Live Load Deflection Performance of Glued Laminated Timber Girder Bridges

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To demonstrate and possibly to promote the increased use of timber bridges in U.S. transportation systems, various agencies, including the United States Department of Agriculture Forest Products Laboratory and FHWA, have supported research to develop improved glued laminated timber bridges. This project is part of this research and identifies acceptable live load deflection characteristics of timber bridges. The relationship between live load deflection and the condition of the asphalt wearing surface is of particular interest. To accomplish this, eight glued laminated timber girder bridges were selected for testing. The performance of the bridges was investigated under live load tests and through bridge inspections. The structures were load tested with fully loaded tandem-axle dump trucks, and global and differential deflection data were collected. Field tests revealed that a significant amount of the asphalt wearing-surface deterioration is the result of differential deck panel deflection.

Timber research and development has contributed significantly to the increase in the construction of timber bridges. Because of the National Timber Bridge Initiative and the Intermodal Surface Transportation Efficiency Act of 1991, funding was made available for timber bridge research. This work is part of that research and is a cooperative effort between Iowa State University and the United States Department of Agriculture Forest Service Forest Products Laboratory. In 1993, AASHTO adopted the load and resistance factor design (LRFD) code for bridges. A global live load deflection criterion of span length divided by 425 is specified. However, this is considered an optional requirement and is left to the designer's judgment. These deflection limits are applied to all material types and bridge types. Consequently, a need exists for design criteria for timber superstructures and decks that is based on actual structural behavior, user perception, and wearing-surface performance.

This paper summarizes the results of the testing and evaluation of eight timber bridges (*1–8*), selected by the Forest Products Laboratory, to identify the relationships among girder deflection, wearing-surface performance, and overall bridge performance. Field inspections were conducted with field tests to investigate the deflection performance of the bridges and the effect on wearing-surface performance. The paper briefly describes the bridges evaluated and their performance under static loading. Table 1 lists all eight bridges along with relevant geometrical and field test information. Observations for limiting differential panel deflection and common factors found to have significantly affected the wearing-surface performance are presented.

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OBJECTIVE

The project scope included field data collection and evaluation under static truck loading and identification of the corresponding effect on the wearing surface and overall bridge performance. The objective for this study was to (*a*) determine the importance of differential panel deflections as they relate to the performance of timber bridges and their wearing surfaces, (*b*) provide relevant information and observations for live load deflection criteria for timber superstructures and decks based on actual structural behavior and performance of wearing surfaces, and (*c*) identify relationships between deflection data and specific deterioration modes and the significance of deflection-induced deterioration.

BACKGROUND

One of the most common types of timber bridges is a glued laminated timber girder bridge with a transverse glued laminated timber deck. These structures vary in width depending on the number of traffic lanes and range in length from 20 ft to greater than 100 ft. These bridges are designed by using deflection criteria based on designer judgment or carried over from steel and concrete bridge design rather than being based on actual structural performance.

Deflection checks for bridges in the United States are evaluated on the basis of deflection criteria typically of the form L/n, where L represents the clear span in inches and n is a constant. The deflection criteria found in *Standard Specifications for Highway Bridges* (*9*), the LRFD bridge design specifications (*10*), and *Timber Bridges: Design, Construction, Inspection and Maintenance* (*11*) are 500, 425, and 360, respectively. In addition, for timber girder bridges, *Timber Bridges* also suggests limiting panel deflection relative to the girders as well as differential panel deflection to 0.10 in. (*11*). Further reduction of the 0.10-in. limit is suggested in the presence of an asphalt wearing surface. No specific limit is given for differential girder deflections, although a limit on effective deck span is presented in *Timber Bridges* to indirectly limit this deflection (*11*).

To investigate the source of the deterioration commonly seen in the wearing surfaces on these types of bridges and the relevance of these deflection criteria, eight bridges were selected, inspected, and field tested. These eight bridges were selected by the Forest Product Laboratory on the basis of their past wearing-surface performance, structural geometry, and location. Data collected from visual inspections

TABLE 1 Tested Glued Laminated Girder Bridges: Geometric, Performance Rating, and Deflection Information

Report Number	Bridge Name/ Year Built	Wearing Surface Rating*	Spans (ft-in.) (tested spans)	# of Girders	Girder Size (in.)	Girder Spacing (in.)	Panel Depth (in.)	Wearing Surface Type	Transverse Cracking (Y/N)	Load Case, Span	Exp. n-value (1995)	Exp. n-value (2003)	Max. Girder Defl.	Max. Differential Panel Defl. (in.)
1	Badger Creek/ ’52	9	30-11	4	8.75 × 30	48	5	Long. planks	N	1	—	110	−0.28	0.022
										2	—	1178	−0.25	
2	Camp Creek/ ’63	7	31-1	4	8.75 × 31.5	48	5	Long. planks	N	1	—	1280	−0.28	—
										2	—	1621	−0.22	
										3	—	2026	−0.16	
3	Lost Creek/ ’74	9	14-3/47/14-3	3	10.75 × 49.5	90	7	Asphalt	N	1	—	2032	−0.27	—
										2	—	2032	−0.27	
4	Erfurth/ ’66	4	40-6	12	14 × 23.5	31	3.25	Asphalt	Y	1	—	515	−0.91	0.127
										2	—	535	−0.88	
5	Chambers County/ recon. ’94	5	51-6	6	8.75 × 43	60	5	Asphalt	Y	1	—	948	−0.64	0.088 (’95)
										2	—	704	−0.85	0.054 (’03)
										3	—	662	−0.91	
										4	816	808	−0.74	
										5	984	968	−0.62	
6	Russellville/ ’94	5	41-9/41-9/ 41-9/41-9	5	6.75 × 41.5	60	5	Asphalt	Y	1	—	1000	−0.47	0.22 (’95)
										2	—	745	−0.64	0.034 (’03)
										3	—	738	−0.64	
										4	1081	1103	−0.43	
										5	929	—	−0.52	
										6	1303	—	−0.37	
7	Wittson/ recon. ’93	6	50/50/102/30	4	6.75 × 43/ 6.75 × 63.25	51	5	Asphalt	Y	1,1	—	60	−0.95	0.117 (Span 1 ’95)
										2,1	1036	996	−0.58	0.027 (Span 1 ’03)
										3,3	938	—	−1.28	0.001 (Span 3 ’03)
8	Butler County/ ’92	2	24/60	5	5 × 27.5	60	5	Asphalt	Y	1	—	796	−0.34	0.176
										2	—	561	−0.48	
										3	—	557	−0.49	

(—) Data not available.
*Deterioration of the wearing surface was rated on a scale from 1–9:
1–severe deterioration of the entire wearing surface
5–moderate deterioration of the wearing surface
9–minor deterioration of the wearing surface.
Recon. = reconstruction; exp. = experimental; defl. = deflection; long. = longitudinal.

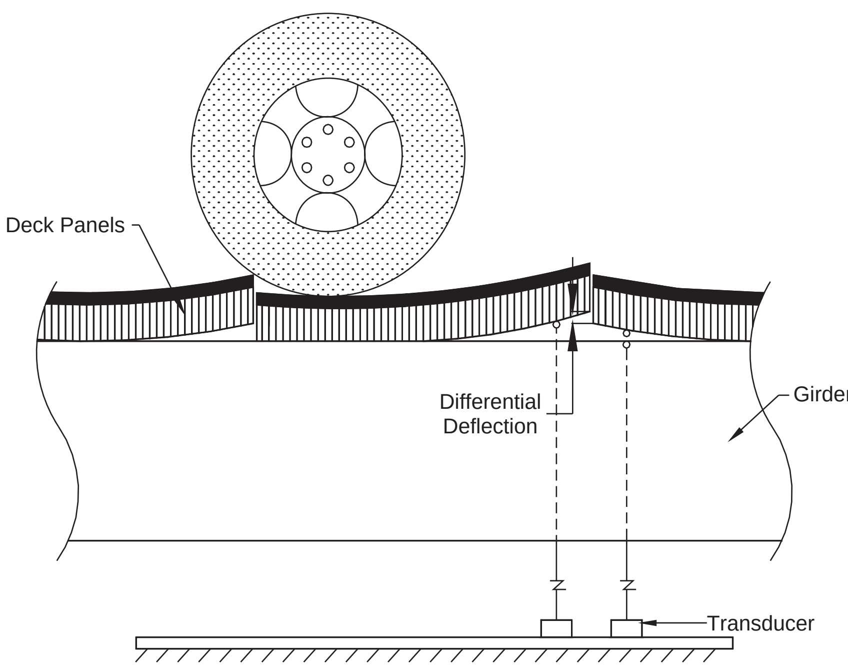
and field load tests conducted in 1995 and 2003 were analyzed and summarized in eight individual reports (*1–8*). The results from this work are the basis for this paper, and the performance of the subject bridges are frequently compared to the preceding criteria.

METHODOLOGY

Field tests involved installing deflection transducers at midspan and quarterspan on both the bottom of the girders and the underside of the deck panels. Transducers were installed on the deck panels such that differential panel deflections could be calculated. A typical instrumentation setup is shown in Figure 1. All global deflections and panel deflections relative to the girders are negative values as they are actually measurements of downward deflection. Differential panel deflections are denoted as positive since they are only a magnitude value and direction has no significance.

Moisture content readings were taken during inspection and are included in the respective bridge reports along with the type of wood preservative. The bridges were loaded with a tandem-axle dump truck moving across the bridge at a crawl speed along several different load paths, and although the test trucks used for each bridge were different, they had similar geometries and load magnitudes. This type of a loading was believed to be more effective at providing useful results than a typical static load placement. In addition, the data were normalized, by total truck weight, to the design truck for comparative

(a)



(b)

FIGURE 1 (*a*) Typical instrumentation setup for deflection measurement and (*b*) instrumentation setup for determination of differential panel deflection.

purposes. By using deflection data collected from these tests and data collected in previous years in conjunction with the condition assessments, conclusions were made on the applicability of current deflection criteria and the effects of live load deflections on wearing surface and bridge performance. Specific comparisons, based on live load deflection and wearing-surface performance, were also made between individual bridges when possible. General comparisons regarding relative deflection magnitudes and wearing-surface performance were also made for all bridges included in this research.

RESULTS

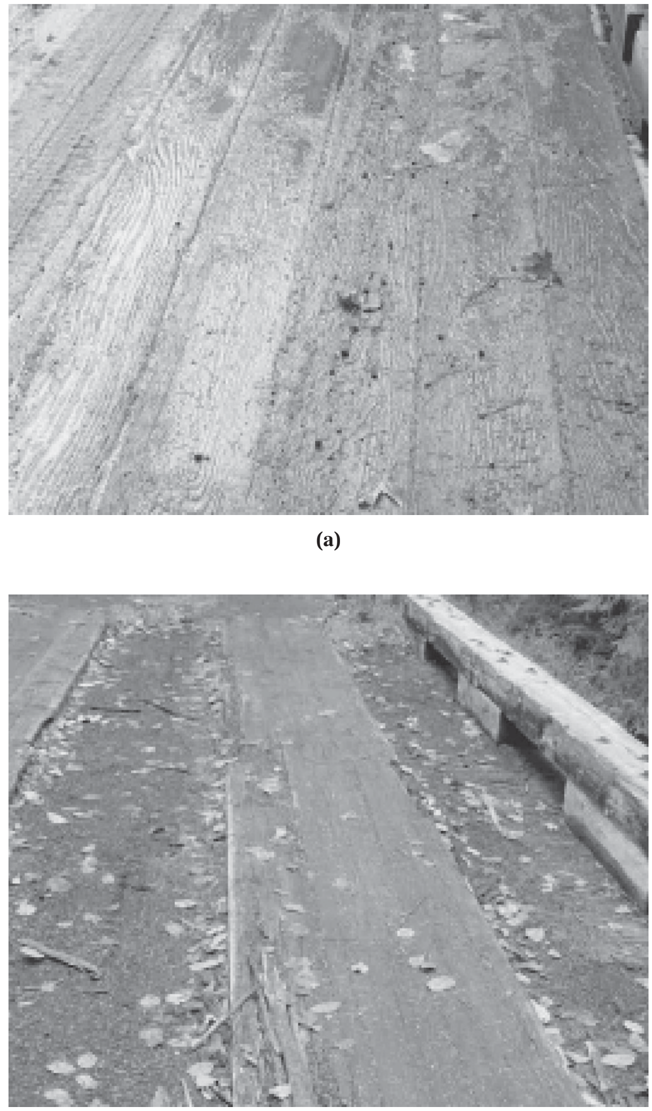
The global and relative deflection performance of each bridge and the condition of its wearing surface related to that performance are discussed. A discussion of the overall performance of the eight bridges and the effects of live load deflection on wearing-surface condition follows.

Longitudinal Plank Wearing-Surface Performance

The Badger Creek Bridge (*1*) spans 31 ft and consists of four glued laminated girders, a transverse glued laminated deck lag screwed to the girders, and a longitudinal timber plank wearing surface (see Figure 2). The bridge showed no signs of wearing-surface deterioration, deflection-induced or otherwise, in the longitudinal plank wearing surface or structural components. The panel joints on this bridge were difficult to locate, and no signs of moisture ingress between the panels were evident; this suggested that there was a tight fit between the deck panels. Similarly, the panels appeared to be well seated on the girders with no visible gaps.

Maximum midspan girder deflections were less than approximately –0.30 in. Panel deflections relative to the girders and maximum differential panel deflections were well within the 0.10-in. limit. (Differential panel deflections were typically less than 0.015 in.) These relatively small differential panel deflections possibly were affected by the longitudinal plank wearing surface, which may reduce differential deflections by distributing load longitudinally from panel to panel. Because of the lack of deterioration in the plank wearing surface, the differential panel deflections and live load deflection behavior of the bridge in general did not appear to be affecting the condition of the wearing surface on the bridge.

The Camp Creek Bridge (*2*) is a single-lane bridge spanning 31 ft and consists of four glued laminated timber girders, a transverse glued laminated timber deck lag screwed to the girders, and three longitudinal planks along each wheel line for the wearing surface (see Figure 2). The remainder of the deck was covered by an asphalt wearing surface. Several uncharacteristic behaviors were evident in the live load deflection of the Camp Creek Bridge. Deflections measured at midspan were typically less than those measured at quarterspan. In addition, the girder and panel deflections follow a stair-step pattern on initial loading, plateau at a peak deflection, and then resumed the stair-step pattern as the deflections decrease. A stair-step deflection pattern refers to the deflections increasing or decreasing for an increment of time, then briefly holding steady, then increasing or decreasing again, briefly holding steady, and so on for numerous cycles or throughout the entire passage of the load vehicle. These behaviors may be caused by transfer of load longitudinally through the timber planks or swelling of the deck panels because of increased moisture content.



(a)

(b)

FIGURE 2 Longitudinal plank wearing surfaces on Oregon bridges: (*a*) Badger Creek Bridge wearing surface (2002) and (*b*) Camp Creek Bridge, deteriorating longitudinal plank wearing surface (2002).

The deflection performance of the structure is within specified limits. Maximum girder deflection for the Camp Creek Bridge, normalized to the design truck, was approximately –0.28 in. Load distribution factors based on the measured deflections were better than predicted design values. Additionally, a basic static analysis found that the support conditions were more like fixed ends than the pinned condition typically assumed in design. This was validated when looking at the support connection detail that likely created a moment couple at the abutment. This also may be a factor affecting the larger deflections at quarterspan than at midspan.

Signs of deterioration were evident in both the longitudinal plank and the asphalt wearing surfaces on this bridge but likely cannot be directly attributed to live load deflections. Rather, the deterioration appears to be from the weather and traffic wear, which were possibly compounded by live load deflections. The basis for this conclusion is that deterioration of the longitudinal planks was evident only in the two lines of planks on the outside and inside of each respective wheel line. As with the longitudinal planks, the deterioration of the asphalt is not believed to be the result of load-induced deflections but is caused mainly by the detachment of the asphalt from the deck panels and wear from traffic and weather. In addition, accumulation of debris on the deck possibly trapped moisture, resulting in the accelerated deterioration of the deck panels and wearing surfaces.

Asphalt Wearing-Surface Performance

Good Wearing-Surface Performance

The Lost Creek Bridge has three spans: a 47-ft main span and two 14 ft 3 in. end spans (*3*). The superstructure is composed of three full-length glued laminated girders, a transverse glued laminated deck lag screwed to the girders and interconnected with steel dowels, a timber sidewalk on one side, and a slight outward taper at one end. Global girder and panel deflections were both within acceptable limits, although a stair-step pattern was evident in the deflection diagrams. The stair-step pattern is believed to be caused by the swelling of the deck panels in combination with the presence of the steel dowels. Maximum differential girder deflection for the main span was approximately 0.13 in. when the load truck was positioned near either the curb or the sidewalk.

Because of complications in the field, differential panel deflections could not be determined. However, panel deflections were calculated relative to the girders and were approximately –0.05 in. Therefore, because of the lack of longitudinal and transverse cracking in the asphalt wearing surface along with the magnitude of the relative girder and panel deflections, the live load deflection behavior of the bridge does not appear to be affecting the condition of the wearing surface on the Lost Creek Bridge.

Moderate Wearing-Surface Performance

The Wittson Bridge is a four-span bridge with variable span lengths, variable depth girders, and a transverse glued laminated timber deck connected to the girders with angle brackets (*5*). One of the two 50-ft spans and the 102-ft span were selected for testing, although fewer data were collected from the long span because of accessibility limitations. Comparison of data collected previously in 1995 with the data collected during testing in 2003 indicates that the deflection performance changed. Maximum girder deflections for Span 1 ranged from –0.50 in. to –1.0 in. in both 1995 and 2003, depending on the load case. The bridge deflections were within acceptable limits for girder deflections for both years.

Panel deflections relative to the girders were approximately –0.03 in. in 1995 and 0.004 in. in 2003. Maximum differential panel deflections on the short span were approximately 0.10 in. in 1995 and 0.03 in. in 2003. Differential panel deflections in 2003 observed for the long span were negligible. The calculated differential panel deflections for Spans 1 and 3 combined with the pattern of transverse cracking suggested that differential deflections may be the source of wearing-surface deterioration. Differential girder deflections between the exterior two girders on Span 3 were approximately 0.30 in.

Significant stair-stepping was evident in the deflection pattern of the girders and deck panels. This behavior and the decrease in differential panel deflections from 1995 to 2003 are believed to be caused by increases in moisture content and the subsequent swelling of the deck panels. The swelling can increase the contact surface friction between the panels. The repeated buildup and release of friction between adjacent deck panels as the load passes over the bridge is likely the source of the stair-stepping behavior.

The Russellville Bridge is also a four-span bridge each consisting of 42-ft spans, five glued laminated girders, and a transverse glued laminated deck lag screwed to the girders (*6*). The bridge is a two-lane structure with transverse cracks along the full length of the bridge at the panel joint locations and only minor longitudinal cracking. The maximum panel deflections relative to the girders were approximately –0.01 in. in 1995 and –0.07 in. in 2003. Maximum differential panel deflection for the bridge was approximately 0.20 in. in 1995 and 0.03 in. in 2003. Swelling of the panels is believed to be the source of the stair-stepping behavior evident in the Russellville deflection data. Girder deflections for Span 1 increased from –0.45 in. to –0.57 in. from 1995 to 2003, and maximum differential girder deflections were typically less than 0.20 in. Only minor longitudinal cracking was evident in the wearing surface.

Poor Wearing-Surface Performance

The Erfurth Bridge (*4*) was selected for evaluation because of its relatively thin (3.5-in.) timber deck. The structure is a two-lane bridge spanning approximately 40 ft with 12 glued laminated girders spaced 31 in. on center and a transverse, glued laminated panel deck consisting of panels 4 ft 4 in. by 3.5 in. The deck panels are attached to the girders with aluminum s-clips on one side of each girder. Each of the 12 glued laminated girders is composed of two separate sections installed side by side. Figure 3 illustrates the condition of the asphalt wearing surface on the Erfurth Bridge at the time of testing in 2003.

The maximum girder deflection and maximum differential panel deflection were –0.85 in. and 0.03 in., respectively. The maximum deflection (–0.85 in.) was measured at Girder G4 in both test cases. Deflection of Girder G5 was approximately –0.40 in. Additionally, inspection of the wearing surface found longitudinal cracks in the asphalt wearing surface above Girder G5. The large differential girder deflection between Girders G4 and G5 is believed to be one cause of the longitudinal cracking in that area (see Figure 3). In addition to the

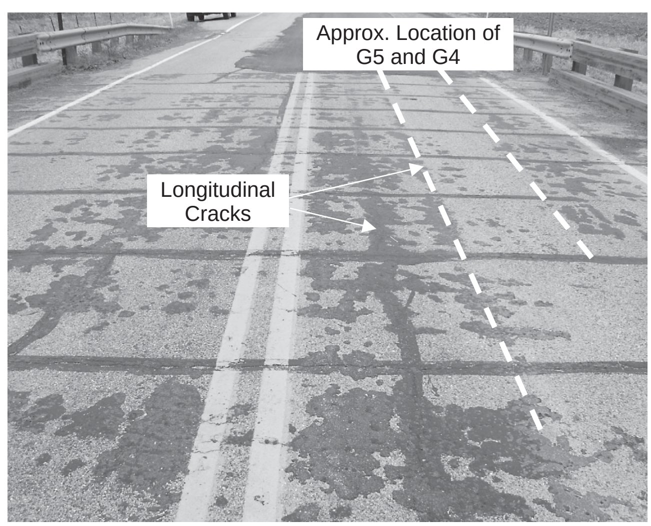


FIGURE 3 Erfurth Bridge, wearing-surface deterioration (2003).

longitudinal cracks above Girder G5, transverse cracks located above each panel joint were evident (see Figure 3). Panel deflections relative to the girders were less than –0.08 in.

The Chambers County Bridge (*7*) is a two-lane, single-span bridge spanning 51 ft 6 in. The bridge consists of six glued laminated girders and a transverse glued laminated panel deck lag screwed to the girders. The maximum girder deflections were approximately –0.65 in. in 1995 and –0.85 in. in 2003. The lack of longitudinal cracking in the wearing surface is the only indication that the differential girder deflections, which were typically less than 0.25 in., are not a significant factor affecting the condition of the wearing surface. However, transverse cracking over the panel joints suggested that relative and differential panel deflections were critical to the performance of the wearing surface. However, the maximum differential panel deflections measured in 1995 and 2003 were 0.08 in. and 0.06 in., respectively. Potholes found in the asphalt wearing surface suggest other factors may have caused the deterioration of the wearing surface, including the asphalt mix design, asphalt placement procedures, and accumulation of debris on the deck, resulting in water retention.

The Butler County Bridge consists of one 24-ft span and one 60-ft span; however, because of access limitations, only the 24-ft span was tested (*8*). The structure is a two-lane bridge consisting of five glued laminated girders and a transverse glued laminated deck lag screwed to the girders. Because of cupping of the deck panels, the largest differential panel deflections occurred when one rear tandem axle was positioned similar to that shown in Figure 4 and the other rear tandem axle was positioned similar to that shown in Figure 4. This configuration produced differential deflections of approximately 0.18 in. Panel deflections relative to the girders were approximately –0.15 in., and both these deflections and the calculated differential panel deflections were greater in magnitude than the recommended limit of 0.10 in. The cupping of the panels not only increased the magnitude of the differential deflections but also increased the number of wearing-surface stress reversals. The cupping of the deck panels was a major

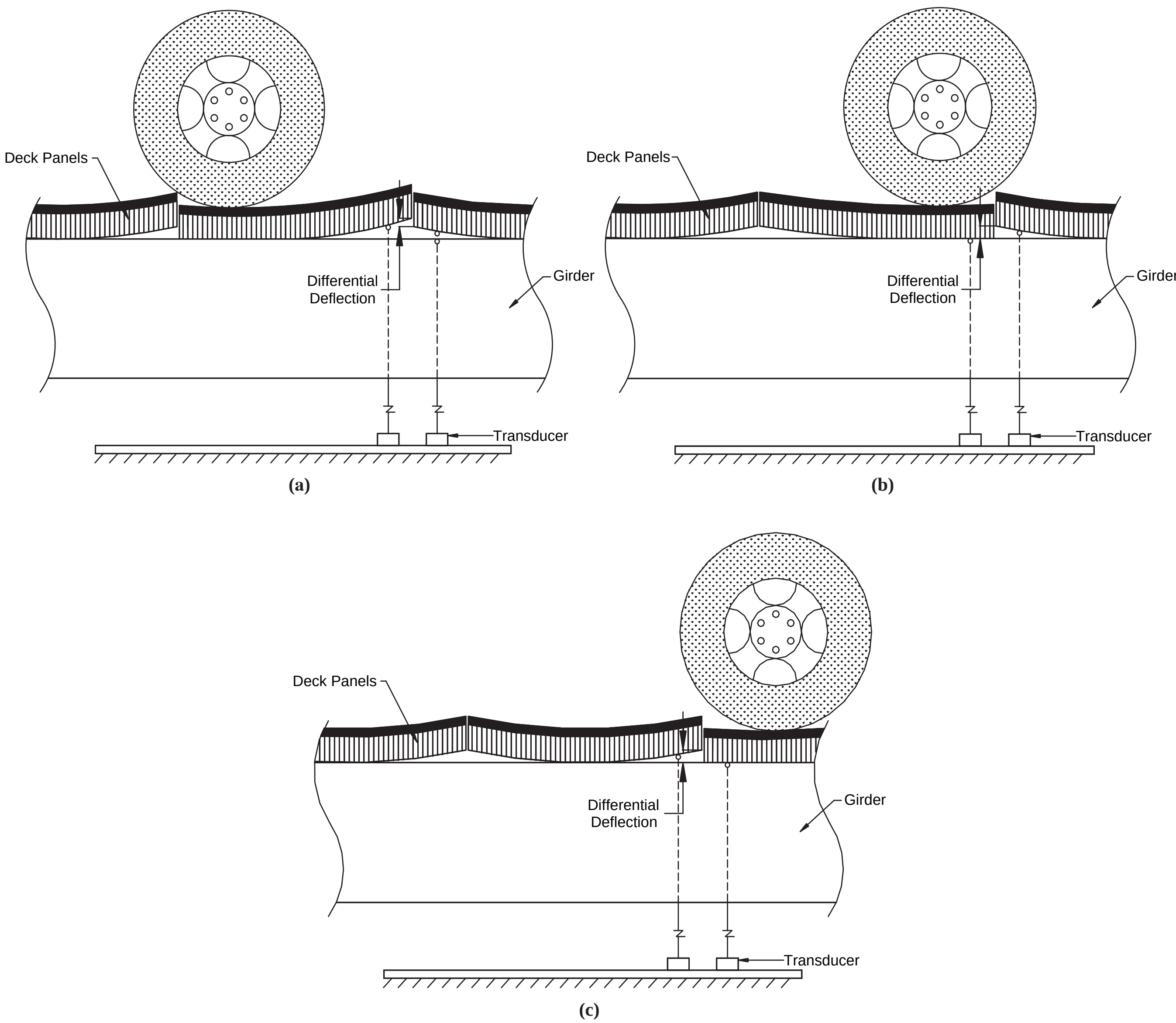


FIGURE 4 Differential panel deflection caused by cupped deck panels.

FIGURE 5 Butler County Bridge, severe wearing-surface deterioration (2003).

contributor to the cracking and disintegration of the asphalt above the panel joints shown in Figure 5.

DISCUSSION OF RESULTS

Global Deflection

On the basis of global girder deflection, the structural performance of the eight glued laminated girder bridges tested for this project was found to be adequate and within recommended limits for all load cases investigated. However, from the collected data and the condition of the deck and other elements, live load deflection is believed to be partially responsible for the deterioration found in the wearing surfaces on these bridges.

The *n*-values calculated for all bridges by using the maximum measured deflections from all load cases are listed in Table 1. Since the deflection criteria are based on the design truck, the experimental *n*-values were normalized for comparative purposes, by total test truck weight relative to the design truck, for that specific bridge.

The large difference between the recommended deflection criteria and that obtained from the experimental *n*-values may be attributed to several factors. The girders may have been initially overdesigned to reduce deflections, or the deflection limit state may not have controlled. Transverse load distribution from girder to girder via the deck panels may be greater than typically assumed in design. In addition, changes in moisture content, support conditions, presence of diaphragms, and other factors may result in smaller deflections than those predicted in design.

Several conclusions may be drawn from the results listed in Table 1 for the five bridges with asphalt wearing surfaces. First, the magnitude of the girder deflections appears to be irrelevant, since girder deflections correlated only to the deterioration of the wearing surface on one bridge, the Erfurth Bridge. However, the girder deflections relative to the span length, or the *n*-values, do provide some useful information. Overall, calculated *n*-values for the five bridges ranged from approximately 500 to nearly 2,000 and differential panel deflections ranged from negligible to just over 0.20 in. The large variance in the *n*-values from one load case to another for an individual bridge is often attributed to the change in position of the load truck from near the longitudinal centerline of the bridge to near the curb. Placement of the load toward the centerline of the bridge allows for the load to be distributed to a greater number of girders than does placement of the load truck near the curbs, resulting in smaller deflections and larger *n*-values. The one structure without transverse cracking in the wearing surface, the Lost Creek Bridge, had *n*-values near 2,000. However, this bridge used dowelled deck panels. The *n*-values for those bridges with transverse cracking were typically lower than 1,200. Third, on the basis of the *n*-values and the differential panel deflections, neither large girder deflection alone nor large differential panel deflections alone appear to be the cause of the cracking seen in the asphalt wearing surfaces. Rather, the combination of large girder deflections with differential panel deflection of generally any magnitude appears to be the controlling factor. However, as mentioned, the asphalt mix design and other factors may also be affecting the transverse cracking seen in the tested bridges.

Differential Deflection

The recommended limit on both panel deflection relative to and midway between two adjacent girders and differential panel deflection is 0.10 in. (*11*). This is intended to be used in addition to global deflection limits, and a reduction in this limit is suggested when asphalt wearing surfaces are used. As noted in Table 1, several bridges exceeded the recommended limit for differential panel deflection but were within the limit for panel deflections relative to the girders. The four bridges with differential panel deflections exceeding the recommended limits (Erfurth, Russellville, Wittson, and Butler County) were all found to have some type of wearing-surface deterioration.

In the case of the Butler County Bridge, the differential panel deflections and significant deterioration of the asphalt were found to caused by the cupping of the deck panels. Typically, the differential panel deflections will spike once during the passage of each load truck axle. Because of the cupping of the deck panels, the differential panel deflections spiked three times for the passage of each axle. This behavior is illustrated in Figure 4. The cupping of the deck panels resulted in multiple stress reversals in the wearing surface for each load that passes over the joint, whereas flat panels typically experience one or two stress reversals per load passage.

Girder spacing on the Russellville Bridge exceeded the recommended limits. This possibly resulted in larger differential panel deflections over the piers and, subsequently, transverse cracking of the asphalt over the piers. The conditions of the wearing surfaces on the Russellville and Wittson Bridges, both four-span bridges, suggested that the two bridges behave differently under live loading. The Russellville Bridge had transverse cracks across the full length of the bridge, whereas the Wittson Bridge had only intermittent transverse cracking across the length of the bridge with no cracking over the piers. Comparison of the time–history deflections for both bridges indicated that both exhibit some continuity across the piers. However, maximum midspan girder deflections for the Russellville Bridge are approximately one-half those from the Wittson Bridge for similar length spans. This is possibly the result of the Russellville Bridge having one more girders than the Wittson Bridge and, therefore, better load distribution characteristics. A major difference between the two bridges is the girder spacing. The Russellville Bridge has a girder spacing of 5 ft; the Wittson Bridge has a girder spacing of 4 ft 3 in., and both bridges have similar-size deck panels. The greater spacing between girders for the Russellville Bridge may produce greater differential panel deflections above the piers, resulting in continuous deterioration of the wearing surface along the full length of the Russellville Bridge and not the Wittson Bridge. The larger girder spacing on the Russellville Bridge was found to exceed the acceptable range for effective deck span specified for this particular configuration (*11*).

In the case of the Erfurth Bridge, the relatively thin 3.5-in. deck panels resulted in the large differential panel deflections. However, in addition to the thin deck, it is noted that large global girder deflections as well as other factors may have effected the deterioration of the wearing surface.

Differential deflections for the two bridges with longitudinal plank wearing surfaces were relatively small. Although differential panel deflections could not be calculated for the Camp Creek Bridge, the similarities in span, girder size, panel size, girder deflections, and load truck compared to the Badger Creek Bridge suggests that differential panel deflections would be similar as well. However, this structure did exhibit some uncharacteristic behaviors for deflection response to loading. These behaviors are believed to be caused by the localized transfer of load longitudinally by the wearing planks, increased stiffness provided by the planks at the interior of the bridge, rotational restraint at the girder ends, and the large curb sections providing additional stiffness to the exterior of the bridge.

Wearing-Surface Performance

To study the relationship between wearing-surface deterioration severity and other bridge characteristics, a scale was created to rate the deterioration of the wearing surfaces. Bridges with transverse cracking at each panel joint measuring 1 in. wide or greater, such as the Butler County Bridge, were rated as a 2. Bridges with transverse cracks at most of the panel joints as well as other minor cracking, such as the Chamber County Bridge, were rated as a 5. Bridges with little to no transverse cracking, such as the Lost Creek Bridge, were rated as a 9. These ratings are summarized in Table 1.

Two types of wearing surface were used on the eight timber girder bridges tested—longitudinal planks and asphalt. The performances of these two different wearing surfaces under live loading were quite different. The longitudinal plank wearing surfaces, used on the Badger Creek and Camp Creek Bridges, performed exceptionally well under live loading. Some signs of deterioration were evident in the longitudinal planks on the Camp Creek Bridge; however, this was determined to be the direct result of the weather conditions and traffic induced wear.

For the most part, the condition of the asphalt wearing surfaces on the other six bridges was moderate to severe, with the exception of the Lost Creek Bridge. The Lost Creek Bridge was the only bridge to have no signs of transverse or longitudinal cracking in its asphalt wearing surface. The other five bridges had significant transverse cracking in the asphalt wearing surface along with minor transverse and longitudinal cracking as well.

Whether the lack of cracking in the wearing surface of the Lost Creek Bridge is caused by small differential panel deflection is unknown since differential panel deflections could not be calculated. The presence of steel dowels connecting adjacent deck panels obviously should reduce the differential panel deflections. As shown in Table 1, the maximum deflections for this bridge are small, with a maximum deflection of $L/2032$, much less than the recommended deflection limit of $L/360$.

The other five bridges with asphalt wearing surfaces showed varied levels of deterioration in their wearing surfaces. In addition, it was found that despite the varied yet acceptable deflection performance of these bridges, the wearing surfaces still showed moderate levels of deterioration. In all cases, some degree of transverse cracking was evident in the asphalt, suggesting that differential panel deflections, although often within limitations, were at least partially responsible.

Severe full-width transverse cracking on the Butler County Bridge was evident at each deck panel joint along the entire length of the bridge (see Figure 5). The transverse cracks were approximately 2 in. wide, and in most cases the moisture barrier between the deck and the asphalt was severed. These cracks were found to be the result of severe cupping of the deck panels, concave upward, which is believed to be the result of changes in moisture content, and may have been compounded by initially being slightly cupped before installation. The cupping of the panels has resulted in gaps between adjacent deck panels of approximately 0.50 in.

In the case of the Wittson Bridge, despite its deflection performance satisfying code requirements, the pattern of transverse deck cracking was atypical when compared to the other bridges tested. Figure 6 illustrates the condition of the wearing surface. Transverse cracks were evident directly over the panel joints for approximately 85% of the length of each of the three shorter spans but were nonexistent 10 ft on each side of the piers (see Figure 6). In addition, minimal transverse cracking was evident over most of the long span.

Similarly, the deflection performance of the Chambers County Bridge was within recommended limits; however, a significant amount of deterioration had occurred in the 3 years since a new asphalt surface was placed in 2000. Judging by photographs from previous inspection reports, similar levels of deterioration were evident in the wearing surface before 2000 as well. Deterioration of the asphalt included transverse cracking above the panel joints, minor transverse cracking, small potholes, and raveling. In addition, the asphalt roadway approaches were significantly deteriorated, creating rough bridge approaches.

On the basis of the measured deflections and the condition of the wearing surfaces for the subject bridges, the performance of single-lane glued laminated timber girder bridges, which use longitudinal timber planks for a wearing surface, is above average. The longitudinal planks appear to have the affect of distributing the load longitudinally from panel to panel, thereby reducing the differential panel

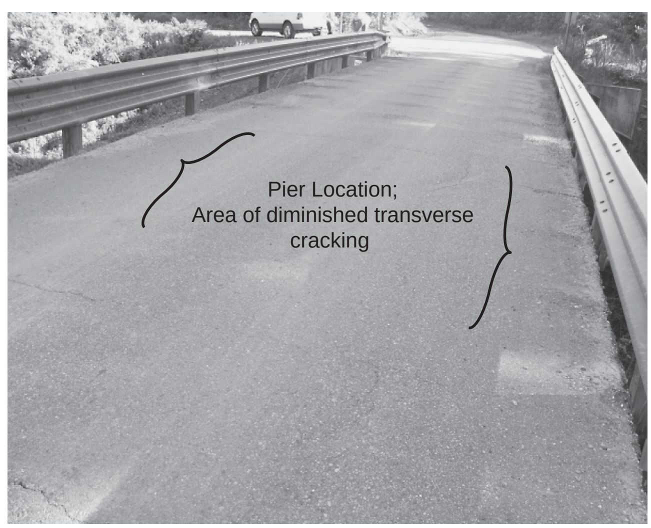


FIGURE 6 Wittson Bridge, irregular pattern of wearing-surface deterioration (2003).

deflections. In addition, the timber planks appear to withstand live load deflections rather effectively. In contrast, the bridges with asphalt-wearing surfaces had varied but significant levels of deterioration in their wearing surfaces. Most of the deterioration was transverse cracking in the asphalt directly above the panel joints. For some bridges, these cracks were along each panel joint; for other bridges, the cracks were over some of the panel joints only. Moreover, the cracks ranged from minor hairline cracks to cracks nearly 2 in. wide. The wearing surface on the dowelled bridge deck performed very well.

OBSERVATIONS

Observations from testing of glued laminated timber girder bridges with transverse glued laminated timber decks and asphalt wearing surfaces are as follows:

- On the basis of the wearing-surface performance, the bridges with higher *n*-values generally performed better. Although it is believed that stiffer bridges may decrease deck deterioration, it is also recognized that any increase in the deflection criteria would result in structural members that provide more load capacity than is necessary and would not be a cost-effective solution to the problem. However, although not cost-effective in the short term, given the significant costs associated with the rehabilitation of bridge overlays, research may be warranted into the long-term cost-effectiveness of these types of structural modifications.
- A stricter limit on differential panel deflection could be considered. Although the magnitude of the differential panel deflections is likely less significant to wearing-surface deterioration than is the repetition of differential panel deflections, a reduction in the limit would likely result in better performing wearing surfaces.
- The variance in the wearing-surface deterioration over the piers on the Wittson and Russellville Bridges appears to be compounded by the larger girder spacing, which exceeds some recommended limits.
- As was clearly shown in the performance of the wearing surface of the Butler County Bridge, the condition of the deck panels (specifically, cupping of the deck panels) was a significant factor affecting the deterioration of the asphalt wearing surface.
- Research is needed to develop inexpensive, construction-friendly, and effective methods to reduce differential panel deflections for both newly constructed and existing structures. Because differential panel deflections can be a significant factor in the deterioration of the wearing surfaces on timber bridges, methods to reduce and remediate differential deflections on both new and existing timber bridges is warranted.
- For low-volume bridges, longitudinal plank wearing surfaces appear to be an effective means of protecting the deck. The dowelled deck panels also appear to be effective.
- Past research has indicated that the design of the asphalt mixes used on these structures may be partially responsible; thus, further research may be necessary into the design of the asphalt mixes used on these types of timber bridges.

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